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Design of Prestressed Wood Bridges Using the Ontario Highway Bridge Design Code

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CONTENTS

1. INTRODUCTION

2. OHBDC SPECIFICATIONS

3. DESIGN EXAMPLE

4. CONSTRUCTION

5. CONCLUSIONS

6. REFERENCES

7. APPENDICES

1. INTRODUCTION

The prestressed wood concept was developed in 1976 as a method of rehabilitating existing nail-laminated wood bridge decks [1]. Using high strength prestressing tendons a permanent pressure is applied perpendicular to the laminations. This creates an artificial confinement and high interlaminate friction. The resulting orthotropic structural medium displays excellent load distribution characteristics, high load sharing strength and a remarkable resistance to the effects of cyclic loading.

Considerable research and development has been performed [2, 3] which has led to the formulation of a set of design specifications on prestressed wood. These specifications have been adopted by the Ontario Highway Bridge Design Code [OHBDC] and will be included in the next edition, which is to be published by the end of 1983.

The objective of this report is to familiarize the designer with the new OHBDC specifications on prestressed wood. This will include both new designs and rehabilitation. Using a design example with direct reference to the specifications enables the discussion of a number of important design considerations.

NOTE: The OHBDC Specifications referred to by this report are up-to-date as of April 1983. The purpose of this reference material is to domonstrate design procedure and interpretation only. The designer should also refer to the new OHBDC specifications which are to be published by the end of 1983.

2. OHBDC SPECIFICATIONS

In order to follow through the design example and subsequent discussion it is necessary to have the latest draft version of the wood specifications and various parts of other sections of the OHBDC. Structural Research Report 83-SRR-2 Parts 1 and 2, [4] contain nearly all of the necessary specifications. It is also recommended that the designer become familiar with that report as it discusses the design of wood bridges in general, using the OHBDC.

A number of specifications which will be needed specifically for the design of prestressed wood bridges, and are not contained in 83-SRR-2, have been included in the Appendix of this report.

3.0 DESIGN EXAMPLE

The following design example involves the design check of the new prestressed bridge shown in Figure 1. This is a 6.0 m simple span structure and only ULS Combination 1 will be considered at the Ultimate Limit State.

Subsequently the prestressing details will be determined according to the requirements of Section 13.22. The details will be determined for the internal prestressing system as shown in Figure 2.

3.1 Ultimate Limit State

Section 13.7 of the OHBDC indicates that longitudinal shear need not be considered for laminated wood decks. Therefore at the ULS only the longitudinal flexural capacity of the deck is considered.

FACTORED MOMENT RESISTANCE

$$M_u = \phi f_{bu} S k_d k_m k_s \qquad (13.22.6)$$

where
- ϕ = performance factor = 0.9 (13.4.4.)
- f_{bu} = specified strength = 11.5 MPa (13.11.2.(a))
- k_d = duration of load effect = 1.0 (13.5.3.)
- k_m = loading sharing effect = 1.5 (13.5.6.)
- k_s = size effect = 1.0 (13.6.2.1.)
- S = section modulus

where
- b = assumed 1000 mm
- d = 290 mm
- $S = \frac{(10^3)(290)^2}{6} = 14.02 \times 10^6$ mm^3

therefore $M_u = 0.9\ (11.5)\ (14.02 \times 10^6)\ (1.5)$

$M_u = \underline{\underline{217.6 \times 10^6}}$ N - mm per m width

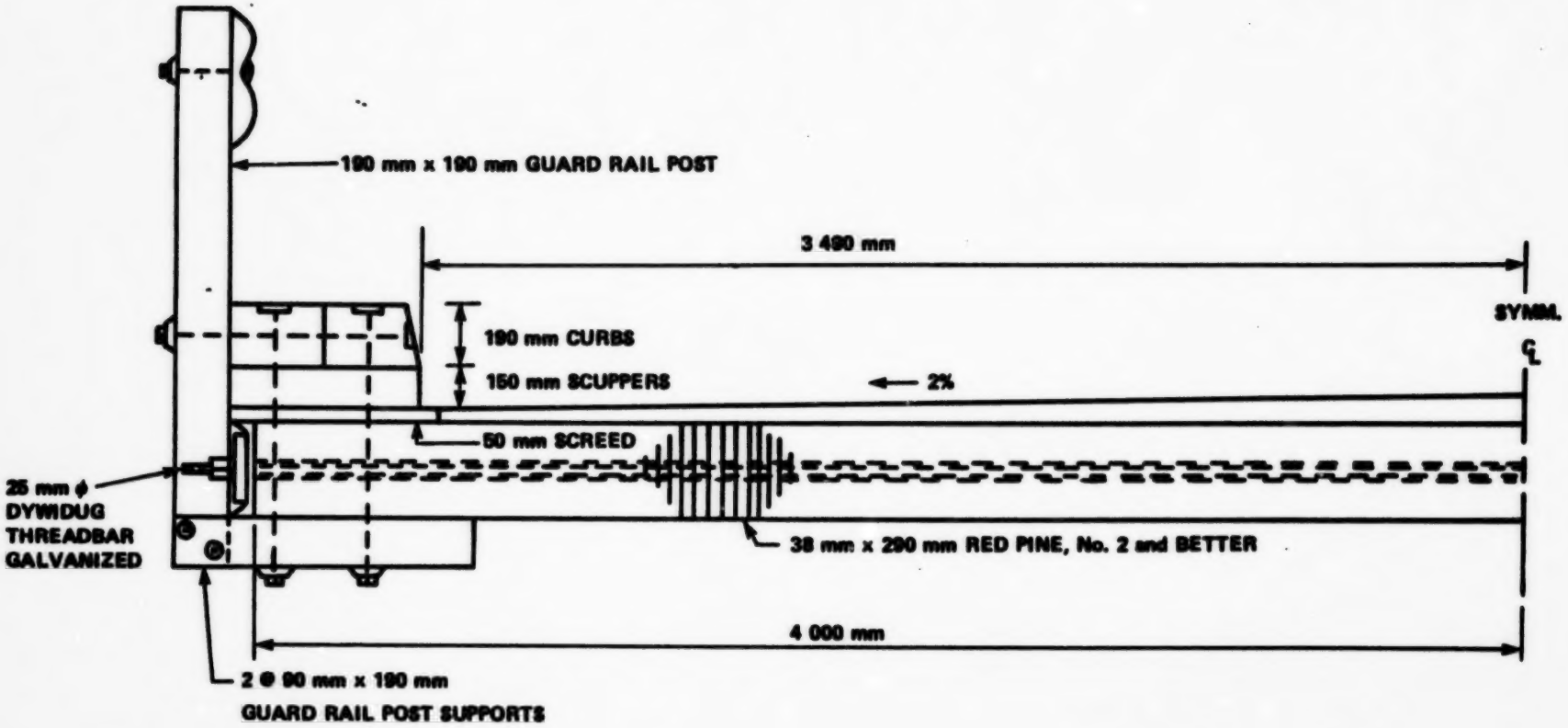

Figure 1/ Design Example

Load Distribution - Simplified Methods

The following additional information is required to apply the simplified method of anlysis in Section 3.7 of the OHBDC.

Design lane width $W_e = W_c/n$ (2.4.3.1.4.)

where W_c = curb to curb width = 6.98 m

$n = 2$ (Table 2.4.3.1.4.)

$W_e = \frac{6.98}{2} = 3.49$ m

Idealization of Structural Parameters (A3.2.(d))

$$D_x = \frac{E_L\, t^3}{12}$$

where $E_L = E_{50} = 9500$ MPa (Table 13.11.2.(a))

$t = 290$ mm

$$D_x = \frac{9500\ (290)^3}{12} = 1931 \times 10^7 \text{ N - mm}$$

$$D_y = \frac{E_T\, t^3}{12}$$

where $E_T = 0.05\ E_L = 475$ MPa (A3.2.(b))

$t = 290$ mm

$$D_y = \frac{475\ (290)^3}{12} = 96.54 \times 10^7 \text{ N - mm}$$

$$D_{xy} = \frac{G_{LT}\, t^3}{6}$$

where $G_{LT} = 0.065\ E_{50} = 617.5$MPa (Table 13.11.2(a))

$$D_{xy} = D_{yx} = \frac{617.5\ (290)^3}{6} = 251.0 \times 10^7 \text{ N - mm}$$

$D_1 = D_2 = 0.0$

Simplified Method (3.7.1.2.1)

$$\alpha = \frac{D_{xy} + D_{yx} + D_1 + D_2}{2(D_x D_y)^{0.5}}$$

$$= \frac{2(251.0)}{2[(1931)\ (96.54)]^{0.5}}$$

$$\alpha = \underline{0.581}$$

$$\theta = \frac{b}{L}\ (D_x/D_y)^{0.25}$$

where b = ½ width bridge = 4.0 m

L = span = 6.0 m

$$\theta = \frac{4.0}{6.0}\ (1931/96.54)^{0.25}$$

$$\theta = \underline{1.41}$$

Select Distribution Factors from Table 3.7.1.2.(a)

D = 2.03 m for external portions

D = 1.68 m for internal portions

C_f = 8.5

Therefore governing D_d value will be;

$$D_d = D\left(1 + \frac{\mu C_f}{100}\right)$$

where $\mu = \frac{W_e - 3.3}{0.6} \leq 1.0$

$$\mu = \frac{3.49 - 3.3}{0.6} = 0.32$$

$$D_d = 1.68\ (1 + \frac{0.32(8.5)}{100})$$

$$D_d = \underline{1.73}\ \text{m}$$

The governing load fraction of one line of wheels

$$\frac{S}{Dd} = \frac{1.0\ m}{1.73} = \underline{0.578} \quad \text{per m width}$$

Live Load "M_{LL}"

Maximum Bending Moment per design lane for a 6 m span is 340 kN - m due to the OHBDC design truck. Therefore the unfactored design load moment for ½ lane is 170 kN - m.

$$M_{LL} = \underline{170}\ \text{kN - m}$$

Dead Load "M_{DL1}" and "M_{DL2}"

Assume 100 mm asphalt for internal portions unit weight of asphalt is

23.5 kN/m^3 (2.4.2.2.)

W_A = 0.10 m (1.0 m) (23.5 kN/m^3) = 2.35 kN/m per m width

W_A = 2.35 kN/m

unit weight of wood is 6.0 kN/m^3

w_w = 0.29 m (1.0 m) (6.0 kN/m^3 = 1.74 kN/m per m width

maximum unfactored dead load moments are;

Asphalt $M_{DL1} = \frac{w_A \ell^2}{8} = \frac{2.35\ (6)^2}{8} = \underline{10.6\ \text{kN - m}}$

Wood $M_{DL2} = \frac{1.74\ (6)^2}{8} = 7.83\ \text{kN - m}$

Dynamic Load Allowance "DLA"

Governing live load is due to two axles, and span is less than 24.5 m therefore the DLA = 0.3 (2.4.3.2.6.). Modified by 0.7 for wood components (2.4.3.2.9).

DLA = 0.7 (.30) = 0.21

Load Factors (Table 2.5.1.(b))

Dead load	asphalt	α_1	= 1.50
	wood	α_2	= 1.20
live + DLA		α_L	= 1.40

Total Factored Load Effect "M_{TOT}"

$$M_{TOT} = \alpha_1 M_{DL1} + \alpha_2 M_{DL2} + \alpha_L \left(\frac{S}{D_d}\right) (M_{LL}) (1 + DLA)$$

$$= 1.5\ (10.6) + 1.2\ (7.83) + 1.4\ (0.578)\ (170)\ (1 + 0.21)$$

M_{TOT} = 191.7 kN - m width

M_{TOT} = 191.7 kN - m < M_μ 217.6 kN - m

therefore okay for ULS

3.2 Transverse Shear at the ULS (13.22.7)

In prestressed laminated wood decks it is necessary to evaluate the transverse shear strength at the ULS, (i.e.: interlaminate friction), due to the prestressing. This will determine the necessary prestress pressure required in order to design the prestressing system.

Factored Resistance to Transverse Shear "V_{tu}"

$V_{tu} = \phi N_f d_\mu$ (13.22.7)

where ϕ = 0.9 (13.22.7)

N_f = to be determined (MPa)

μ = 0.35 (dressed material) (13.22.7)

d = 290 mm

therefore $V_{tu} = 0.9(290)\ (0.35)\ (N_f)$

$V_{tu} = \underline{91.35\ (N_f)}$ N/mm

Load Distribution

Only live load and DLA are considered for transverse shear at the ULS. The distribution of load is determined by the simplified method of analysis as outlined in 3.7.1.7.

$$\beta = \pi\ (2b/L)\ (D_x/D_{xy})^{0.5}$$

where b = ⅓ bridge width = 4.0 m

L = 6.0 m

D_x = 1931×10^7 N - mm

D_{xy} = 251×10^7 N - mm

$\beta = \pi\,(2(4.0)/6.0)\quad (1931/251)^{0.5}$

$= 11.62$

using β the unfactored transverse shear intensity is selected by interpolation using the charts provided in Fig. 3.7.1.7.

(Note: A new chart for torsionally weak decks is to be added to the figure before publication, a copy of this figure is given in the Appendix to this report).

By interpolation the unfactored transverse shear intensity is

22.0 kN/m = 22.0 N/mm

The total factored shear intensity "V_{TOT}" is

$V_{TOT} = \alpha_L\,(22.0)\,(1 + DLA)$

$V_{TOT} = 1.4\,(22)\,(1 + 0.21)$

$V_{TOT} = \underline{37.3}$ N/mm

Therefore in order to satisfy the ULS for transverse shear

$V_{TOT} < V_{tu}$

$(91.35)N_f > 37.3$

$N_f > \dfrac{37.3}{91.35} = 0.41$ MPa

The required minimum prestress pressure, after losses must be > 0.41 MPa to satisfy the ULS.

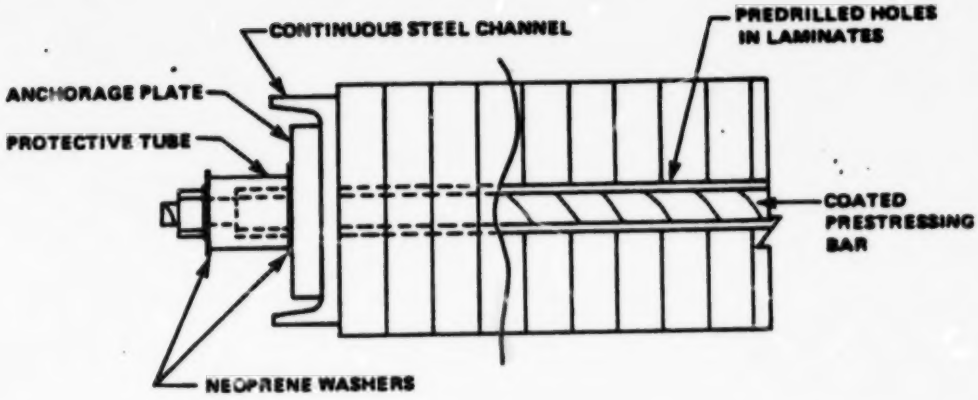

FIG. 2. PRESTRESS DETAIL FOR NEW DESIGNS

3.3 Design of Prestressing System

The minimum final prestress pressure N_f must be

$$N_f = 0.410 \text{ MPa} > 0.35 \text{ MPa} \quad (13.22.3.2.), \quad \text{okay.}$$

Long term losses due to creep are 50% (13.22.3.2.)

therefore minimum initial prestress pressure N_i

$$N_i = \frac{N_f}{0.5} = \frac{0.41}{0.5} = 0.82 \text{ MPa}$$

Assume 5% losses for anchorage slip (see appendix). Ignore effects of relaxation of prestressing steel.

Minimum prestress at jacking N_j

$$N_j = \frac{0.82}{0.95} = 0.863 \text{ MPa}$$

Maximum prestress pressure allowed $0.25f_{q\ell}$ (13.22.3.2.)

$$N_j = 0.863 \text{ MPa} < 0.25f_{q\ell} = 0.25(5.0) = 1.25\text{MPa} \quad - \text{ okay.}$$

3.3.1. Internal Prestressing Steel System.

Assuming an internal prestressing system (see Figure 2).

Select 25 mm ϕ Dywidag Threadbars

Material specifications $f_{pu} = 1034$ MPa

$A_{ps} = 548 \text{ mm}^2$

Try maximum spacing of prestressing bars (13.22.4.4.3.)

$6d \leq 1.5$ m

$6d = 6(0.29 \text{ m}) = 1.74$ m

therefore use spacing $s = \underline{1.5 \text{ m}}$

Check Steel/Wood ratio (13.22.3.1.)

$$\frac{A_{ps}}{A_w} \geq 0.0016$$

where A_{ps} = area prestressing steel = 548 mm^2

A_w = area wood prestressed = s (d)
by one bar

A_w = 1500 mm (290 mm) = 435000 mm^2

$\frac{A_{ps}}{A_w} = \frac{548}{435000} = .00126 < .0016$ okay

Check <u>stress in prestressing steel at jacking</u> f_{pj} <u>(13.22.2.4.)</u>

$$f_{pj} = \frac{N_j(A_w)}{A_s} = \frac{0.863 \text{ MPa } (435000)}{548 \text{ mm}^2}$$

f_{pj} = 685 MPa

which is $\frac{f_{pj}}{f_{pu}} = \frac{685}{1034} \times 100\% = 66\%$ of f_{pu} - okay.

3.3.1.1. Anchorage Details

Anchorage details must satisfy <u>(13.22.4.2.)</u>

<u>Factored Bearing Resistance</u> R_u

$R_u = \phi f_{q\ell} A_b$ (13.22.4.2.)

where ϕ = 1.0 (13.4.4.)

$f_{q\ell}$ = 5.0 MPa

A_b = direct bearing area. To be determined.

$R_u = 1.0\ (5.0)\ A_b = 5.0\ (A_b)$

also $R_u > P_j$ prestressing force at jacking

where $P_j = A_w\ (N_j) = 435000\ (.863)$

P_j = 375 400 N

therefore $5.0(A_b) > 375\ 400$ N

$A_b > 75080\ mm^2$

Steel Channel (13.22.4.4.)

depth channel $d_c > 0.8d = 247$ mm

moment of inertia about weak axis $I_c > 140\ 0000$ mm^4

web thickness $W_c > 11$ mm

Select channel 254 mm @ 45 kg/m

where $d_c = 254$ mm $\nmid d$ okay

$I_c = 164\ 0000$ mm^4

$W_c = 17$ mm

Anchorage Plate (see Figure 13.22.4.4.1.(b))

Width of plate w to fit into channel

maximum $w = 200$ mm

length of plate L_p must satisfy.

$$1.0 < \frac{L_p}{w} < 2.0$$

also L_p must satisfy earlier requirement for A_b

$A_b > 75\ 080$ mm^2

where $A_b = b_b\ (L_b)$ (13.22.4.4.4.)

and b_b = depth of channel = $d_c = 254$ mm

$L_b = L_p + 2\ (W_c) = L_p + 2\ (17)$

$A_b = 254\ (L_p + 34)$

therefore

$A_b > 75080$ mm^2

$254\ (L_p + 34) > 75\ 080$ mm^2

$L_p > 262$ mm

try $L_p = 300$ mm

$\frac{L_p}{w} = \frac{300}{200} = 1.5$ okay

thickness of plate t_p

$t_p > \frac{L_p}{12} = 25$ mm

use plate 200 mm x 300 mm x 25 mm

therefore final R_u

$R_u = \phi f_{q\ell} A_b = 1.0\ (5)\ (254)\ (300 + 34)$

$R_u = 424\ 200$ N

Stress prestressing to maximum to offset effects of creep.

$P_j = R_u$

Therefore $f_{pj} = \frac{P_j}{A_{ps}} = \frac{424\ 200}{548} = 774$ MPa

which is 75% of f_{pu} okay

and $N_j = \frac{P_j}{A_w} = \frac{424\ 200}{1500(290)} = 0.975$ MPa < 1.25 MPa okay

Summary

25 mm ϕ Dywidag Threadbars @ 1.5 m spacing

254 mm Channel @ 45 kg/m

200 mm x 300 mm x 25 mm Anchorage Plates

Force per bar at jacking 425 kN

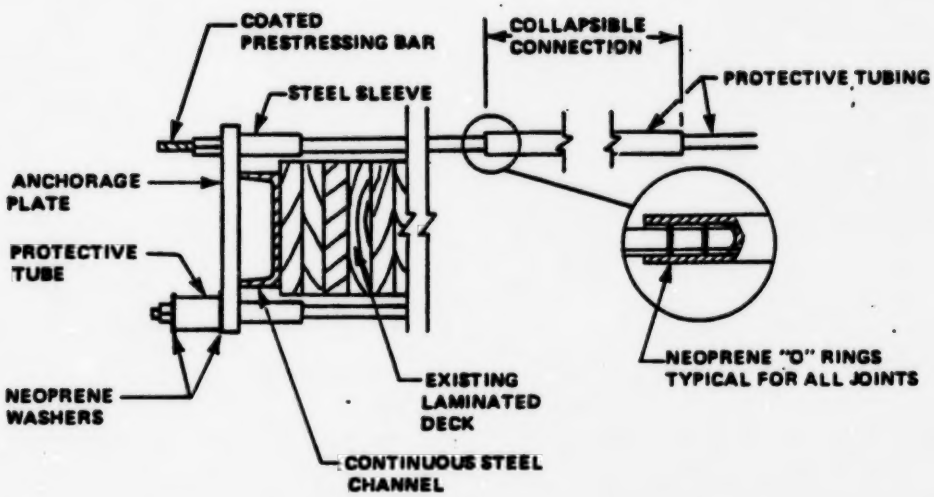

FIG. 2. PRESTRESS DETAIL FOR REHABILITATION

3.3.2 External Prestressing System (Figure 3)

Basically the same procedure is applied to determine the necessary prestressing details for the external system. Except here we use two bars per station and the distribution of direct bearing, due to prestress, is different (13.22.4.4.4.). In addition, this anchorage detail requires web stiffening of the channel (13.22.4.4.5.) in order to ensure proper prestress transfer.

3.4 Serviceability Limit State I (SLSI)

Maximum Allowable Deflection "Δ_A"

$$\Delta_A = \frac{1}{400} \text{ of the span} \qquad (13.4.2.)$$

$$\Delta_A = \frac{6\,000}{400} = 15.0 \text{ mm}$$

Live Load Deflection (SLSI) "Δ_L"

Simplified Method of Analysis (3.4.5.4.) & (3.7.1.2.2.)

Determine new D_d value for one lane loaded using previous α and θ values.

$$\alpha = 0.581$$

$$\theta = 1.41$$

from figure 3.7.1.2.2.(b)

$$D = 2.03 \text{ m} \qquad C_f = 3.0$$

$$D_d = D\left(1 + \frac{\mu C_f}{100}\right) = 2.03\left(1 + \frac{(0.32)\ (3.0)}{100}\right)$$

$$D_d = 2.05 \text{ m}$$

Section 3.7.1.2.2. allows for a modification which reduces the load effects when the outside line of wheels is more than 1 m from the edge of the bridge. The SLSI requirement requires the vehicle to be placed in the centre of the driving lane so this modification is usually applicable. However, for a first check, since deflection does not usually govern smaller spans, use the D_d as derived above.

The deflection is determined by applying one line of wheels of the SLSI, without DLA, to a width D_d of the bridge.

Δ_L* = 12.0 mm < A_A = 15.0 mm okay

* Calculations not included. Two ±40 kN axles governed.

4.0 Construction

The designer is at liberty to design any form of anchorage and prestressing details that meet the requirements of Section 13.22 of the OHBDC. However the steel channel bulkhead details provided in the Code have proven very successful in the field. Some local crushing of wood will occur beneath the channel at the anchorages but it is not visible without very close scrutiny.

4.1 New Construction

Several new prestressed wood decks have been constructed, the latest of which was the 120 m long continuous deck on the Sioux Narrows bridge in 1982 [3]. The deck was designed using 38 mm x 190 mm x 6 m Douglas Fir with 38 mm ϕ holes for 25 mm ϕ bars. In order to enhance the line-up of prestressing holes, 6 mm gaps were introduced at each butt-joint. The Contractor reported no problems in constructing the deck or installation of the prestressing bars.

It is necessary to pre-plan the laminate arrangement, including the location of prestressing holes. All lumber must be cut and drilled before pressure preservative treatment. In the case of the Sioux Narrows deck 95 percent of the material was supplied the same length and contained symmetrical hole locations. Only the starting and ending laminations were of different length and required orientation. It is at the design stage where simplification of the construction stage can be implemented.

Prestressed decks cannot be anchored to the supports until the primary prestressing has been performed. Otherwise the effects of prestressing would be resisted, as the deck tends to move considerably in the initial prestressing stages. This movement, due to elastic compression and the closing of spaces between laminates, is usually less than 1%. Therefore the design width of the bridge should accommodate for at least a 1 percent reduction in width after prestressing.

Prestressed decks cannot be constructed to provide cross fall or a changing super elevation. Should this be the case, the deck will straighten out during prestressing and lift off the supports. Even in perfectly level decks some so-called "curling" occurs during prestressing, but this is not serious.

In some areas, it may be necessary to apply an external load on the deck to make it rest on the support for anchoring. Usually this requires a very small load and can be done using a construction vehicle and/or reducing the prestress in the deck. However this is unusual, except where a new prestressed deck is being applied to an existing bridge where the substructure may not be level.

It is difficult to determine the necessary capacity of the support anchorages as they may have to resist components of the prestressing forces. As a general rule of thumb it is suggested each anchorage unit have a capacity of at least 50 percent of the prestressing force at one station. In addition the spacing of anchorages along the support should not exceed 3 m, but not be less than three anchorages at each support. The drawings should allow flexibility of installation location in the event that uplift occurs. This requires that the anchorages be of a type which can be located anywhere along the supports, except possibly where the pile cap rests on a pile. A successful anchorage arrangement has been used several times in the field and consists simply of pairs of threaded rods, with steel end plates. These are used to form a clamp through the deck and about the pile cap. Any holes drilled through the deck must be treated according to 13.16.5.

4.2 Rehabilitation

Basically the same conditions apply to rehabilitating existing bridges by prestressing. However the designer must provide the additional environmental protection specified in 13.22.8.1.

The spacing of prestressing stations on an existing bridge may not be uniform. In fact it is advantageous to reduce the spacing between the stations directly on either side of the supports, by as much as 30 percent. The prestressing force should not be reduced immediately in these stations as this additional pressure on the wood, though it may exceed the requirements of 13.22.3.2., helps to break the original support connection which was provided by toe-nailing of the laminations to the support. However site observation during prestressing should be performed and may result in reducing the force in these stations should any visible crushing occur.

It will not be possible to maintain alignment along the edge of an existing deck. The deck will always narrow towards the middle of the span and bulge out over the supports, where the original support connections resist movement.

Existing decks will nearly always display uplift of the supports near the outside edges of the deck. The same anchorage requirements should be applied to these decks as for new decks, except that at least 50% more anchorage units should be supplied. Flexibility in installation location is necessary, but at least three units per support should be installed for a two lane bridge and four for decks up to 15 m wide. Prestressing of an existing deck exceeding 15 m is not recommended without due attention to the possibility of buckling of the deck.

Installation of prestressing on an existing deck is best conducted with the removal of the existing wearing surface. On some existing bridges troughs have been cut into the asphalt to receive the top prestressing bars, but the reconstruction time was increased considerably. If the bridge cannot be closed during reconstruction then a two directional plank running surface for traffic is recommended, which allows the installation of bars beneath the traffic.

4.3. Jacking

The MTC has assembled a multi-jack hydraulic system including twenty-four 540 kN hollow hydraulic cylinders. The system includes all the necessary steel hardware to perform prestressing of both the internal and external systems. It also provides access to the anchorages for advancement during prestressing. This hydraulic system is available for rental by contractors or other government organizations throughout Ontario.

CAUTION is advised when prestressing high strength steel bars. The geometries of wood structures are not as stable as concrete or steel and so strict field supervision is recommended. There are eight prestressed wood bridges presently in Ontario and no prestressing failures have occurred during construction. However, two failures occurred on one structure where the proper coupling length was not achieved during the restressing. Only minor damage to the bars resulted as MTC practice allows no one in the vicinity of the anchorages during stressing.

5. Conclusions

The design of prestressed wood as outlined in the OHBDC is based, for the most part upon research and developement performed by or for MTC. The exprience gained in the field applications has been applied to the developement of the suggested anchorage details.

The most important design requirement for the prestressing system is the steel/wood ratio (13.22.3.1). It is this requirement that ensures adequate flexibility to offset the effects of creep in the wood. The value 0.0016 should be considered as an absolute maximum and where possible a lower value should be used.

Exceeding the prestress limitations on the wood ($N_i > 0.25f_{q1}$) will not affect the structural strength of the deck as long as it is not exceeded by too much. However, it will not benefit the losses due to creep. Only within the range of prestress levels specified is the 50 percent losses applicable. If the $0.25f_{q1}$ is exceeded then the maximum final prestress level should be taken as 50 percent of $0.25f_{q1}$.

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7. Appendices

Determination of losses due to anchorage slip

In a prestressed wood deck the stiffness of both the wood and the steel must be considered. The prestressed system may be simulated by taking both elements in series as shown below;

The overall displacement under a load P can be expressed as D

Where $$D = \frac{P\,L}{A_w E_w} + \frac{P\,L}{A_s E_s}$$

therefore the force in the bar would be;

$$P = \frac{D}{L} \quad \frac{1}{\frac{1}{A_w E_w} + \frac{1}{A_s E_s}}$$

Where L = length of bar or width of the deck = 8 000 mm

A_w = area of the wood stressed by one bar = 435 000 mm^2

A_s = area of prestressing steel = 548 mm^2

E_w = modulus of elast. of wood perpendicular to the grain = $0.05\ E_{50}$ = 475 MPa

E_s = modulus of elast. of steel = 200 000 MPa

D = internal displacement (anchorage slip)

Therefore if we introduce 3 mm of anchorage slip the equivalent prestressin force would be ;

$$P = 26\,855\ N$$

with P = 424 200 N this represents a 6 percent loss.

determined. For values of θ and δ other than those shown in Table 3.7.1.6.1, linear interpolation may be used in order to determine the value of λ_s.

3.7.1.6.2/ Multi-Spine Bridges

(a) For multi-spine bridges of concrete construction or of steel construction with at least three spines, and having a cross-section which satisfies the requirements of Clause 3.7.1.5.2(a), the method of Clause 3.7.1.5.2(a) may be used, provided that the value of D_d obtained by that method is reduced by 10%.

(b) For multi-spine bridges of steel-concrete composite construction, the method of Clause 3.7.1.5.2(b) may be used, provided that the load fraction L_L is determined by the following equations:

$$L_L = 1.05W_L$$

for ultimate limit state and serviceability limit states type II,

$$L_L = 0.60W_L$$

for serviceability limit states type I.

3.7.1.7/ Transverse Shear in Multi-Beam Bridges

The maximum intensity of transverse shear shall be assumed to occur when there is only one design vehicle on the bridge.

The following simplified method may be used:

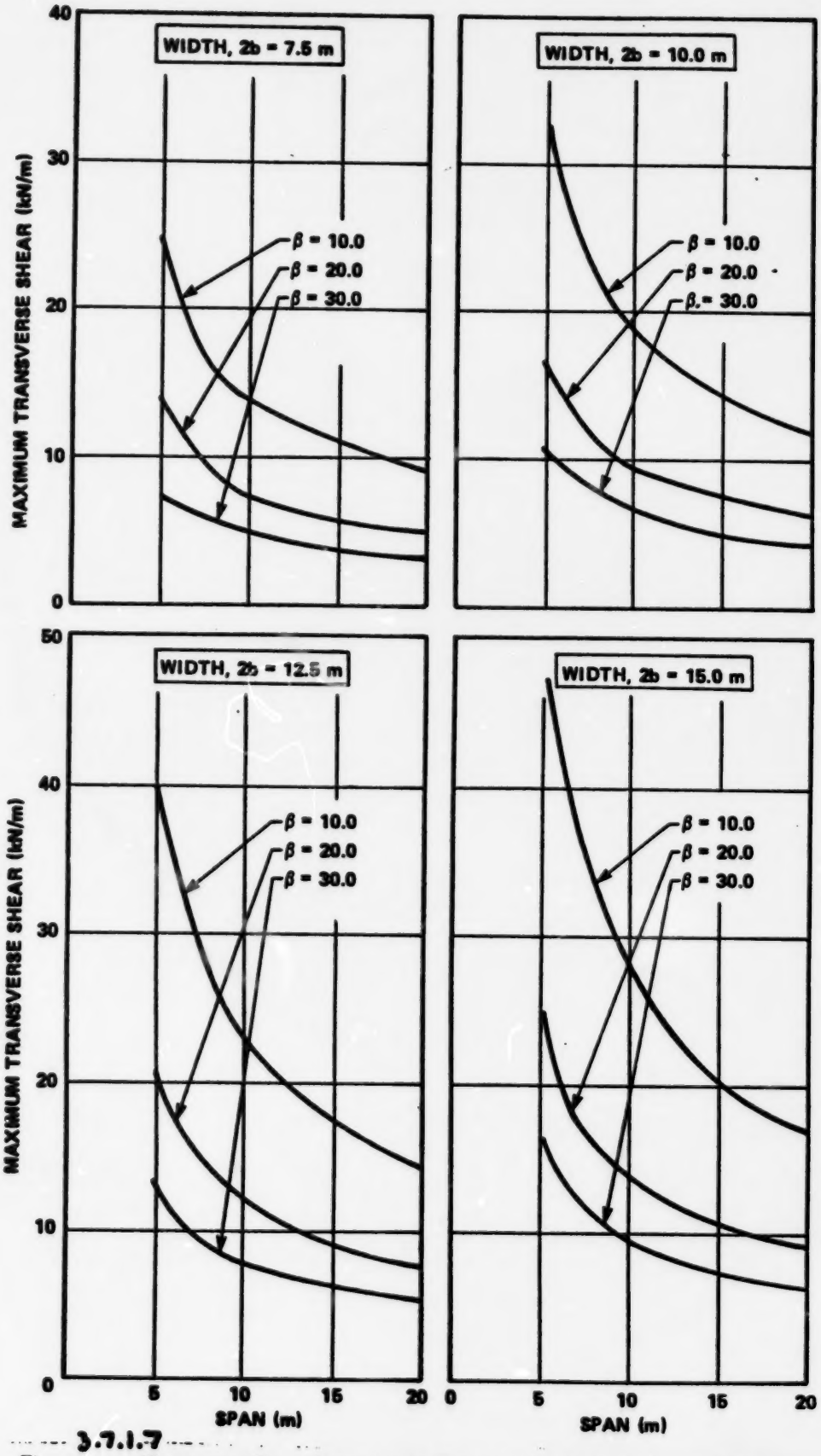
(i) Calculate $\beta = \pi\left[\frac{2b}{L}\right]\left[\frac{D_x}{D_{xy}}\right]^{0.5}$

(ii) Using the charts of Figure 3.7.1.7 for the appropriate values of span and β, obtain the value of transverse shear intensity. Use linear interpolation for this intensity in the case of a width falling between the widths given on the charts.

3.7.1.8/ Analysis of Transverse Beams

In analysing transverse beams covered by a deck, the effect of a concentrated load situated between two parallel adjacent transverse beams may be calculated as follows provided that there are no stringers or, if stringers are present, the decking is continuous over the stringers.

(i) The original concentrated load shall be replaced by statically equivalent concentrated loads on the transverse beams, acting at



3.7.1.7

Figure Transverse Shear Intensity in Torsionally Weak Articulated Plates due to OHBDC Design Truck